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DISCUSSIONS ON DYNAMIC INTERACTION BETWEEN PILES AND LARGE PARTICLE ROCKFILL

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ABSTRACT

A series of full-scale laterally loaded pile tests were conducted at the University of California, San Diego in 2007 in order to obtain a better understanding about pile-rockfill interaction for seismic design of port facilities. The project was composed of three experiments where the cyclic lateral loads were cyclically applied at the heads of the piles fully instrumented using a hydraulic actuator. Comparing the experimental and numerical results, assessments of p-y curves used for current design practice were performed. As a result, it was found that the soil pile springs currently used for design gave much lower lateral resistance than recorded in the experiments. This series of the experiments could provide very useful information for deformation based design, and effects of loading rate and type on behavior of pile-soil system still need to be considered to develop a more sophisticated seismic design. In the first part of this paper, a brief description of the experiments and some examples of the test results are presented. Based on the test results and the observations during the tests, some of possible factors affecting soil-pile interaction under dynamic cyclic load conditions are discussed in the second part.

INTRODUCTION

Nowadays, performance based design has become more familiar in design codes for many types of structures; port facilities are not an exception. The recently approved seismic code for Port of Los Angeles (POLA) [2004] requires estimation of the performance for two levels of earthquake intensity. For the case of pile-supported wharf structures, the expected performance is usually derived based on the estimation of the structures' displacement capacity.

Soil-pile interaction is one of the key factors which needs to be clarified in order to develop more reliable design methodologies and parameters. Piles for wharf structures are frequently driven into large particle size rockfills. There is a relatively large amount of information available about behaviors of full-scale piles embedded in typical soils, such as sand and clay, but limited information is available discussing the behavior of piles in large-diameter rockfill materials.

In order to fill this knowledge gap, a series of full-scale load experiments on piles in large particle rockfill were conducted at the Soil-Foundation-Structure-Interaction Test Facility at the University of California, San Diego's Englekirk Center in order to validate and improve our understanding of the seismic performance of wharf-pile-rockfill dike system.

In the first part of this paper, the test setup and instrumentation are briefly presented. Noticeable observations during and after test are described, followed by consideration on possible mechanism of reaction generation in the second part. Thirdly, reaction displacement curves, *p-y* curves for large particle rockfill used for current design practice was verified based on the test results. Finally, some factors which may significantly affect pile behavior are discussed to extend knowledge obtained in this study to seismic design or estimation of dynamic behaviors of wharf structures in rockfill.

PAST WORK REVIEW

The current POLA seismic code provides reaction displacement curves, *p-y* curves, for large particle size rockfill material used for design of pile-supported wharf structures (e.g. Martin, 2005). These *p-y* curves have been based on previous research.

Diaz *et al.* [1984] conducted experiments using instrumented octagonal piles placed directly at a port construction site. A monotonic load was applied to the test pile tops, and load displacement curves at the pile top and moment profiles along the piles could be obtained in addition to *p-y* curves. However, because this set of tests was conducted with sparse instrumentation, the available information can not easily be quantified. In addition, the test piles were located close to the dike crest and pushed only toward the downslope, thus no information was available for the response of piles on level ground.

Extensive research was carried out by McCullough *et al.* [2001] using a centrifuge in order to understand the dynamic behavior of pile supported wharves. Gradation of gravel used was scaled based on scaling low. However, because of shortage of large- and full-scale test results, the centrifuge results with scaled specimen are still difficult to be verified.

Based on the past work review above, additional full-scale tests are obviously needed in order to obtain supplemental data and a more complete understanding of the interaction between wharf piles and large grain-size rockfills.

TEST SETUP AND INSTRUMENTATION

Five prestressed concrete octagonal piles were constructed and fully instrumented. The test piles and the pile-load stub connection had the same properties as piles and pile-deck connection currently used in POLA wharves, respectively. The piles were arranged in three test sets with different boundary conditions and dimensions as shown in *Table 1* and *Fig. 1*. The pile for the single pile test was located 3.66 m (six times larger than the pile diameter) from the slope crest of the quarry-run (P1 in *Fig. 1*), and its fixity condition at the pile top was free. For the coupled pile tests, one of the test piles was installed 3.66 m from the crest and the other was located at the crest (P3-P5 and P2-P4). Also, the test piles were interconnected with a fully instrumented steel beam and the fixity of the pile top was partially fixed because the connection allows some rotation. Section views for Single pile and Coupled pile tests are shown in *Fig. 2* and *Fig. 3*, respectively.

In construction, the embankment and dike are constructed first, and then the piles are driven into rockfill in the design depth by vibration or jet methods. However, it is not appropriate to follow the same procedure for these tests because pile driving into rockfills requires a large, powerful device and may damage the instrumentation preinstalled in the piles. Therefore, the piles were placed inside a previously excavated soil pit, and the rock then vibrated using a vibratory roller to simulate effects of pile driving. Since the final pile locations were critical for proper actuator placement, a concrete footing was built at the bottom of the piles to fix the locations during backfilling.

Table 1 Summary of test sets

Test name	Test piles	Boundary condition at the top of piles	Clear space between bottom of cap and ground surface
Single pile	P1	Free	1.07 m
Coupled pile 1	P3 & P5	Fixed	1.07 m
Coupled pile 2	P2 & P4	Fixed	1.68 m

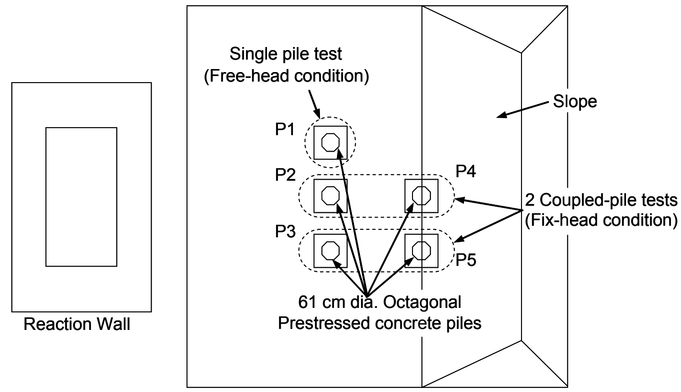


Fig 1. Test setup (Plan view)

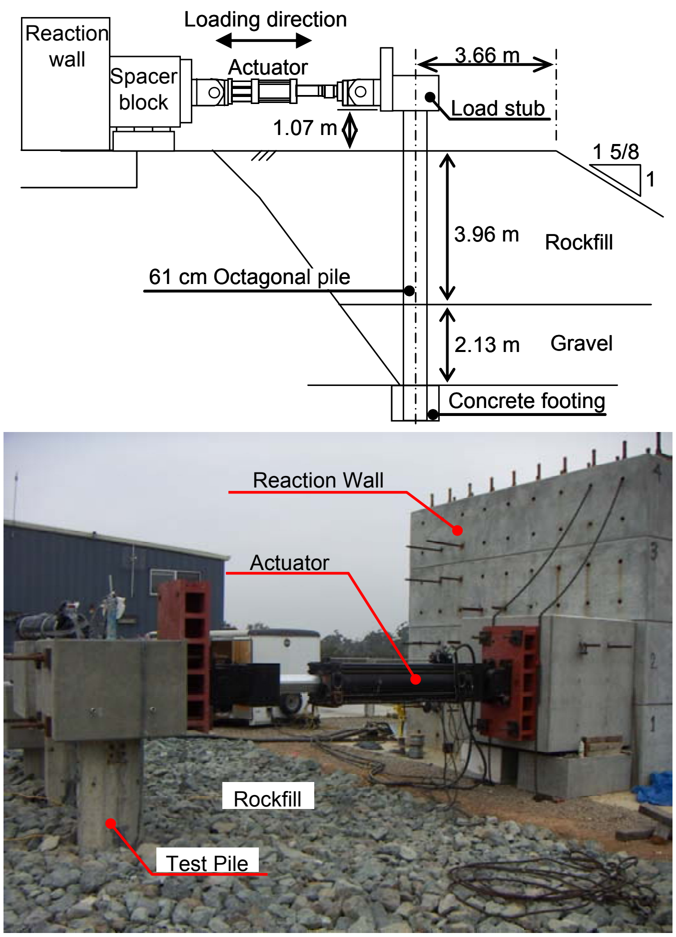


Fig. 2 Test setup – Single pile test

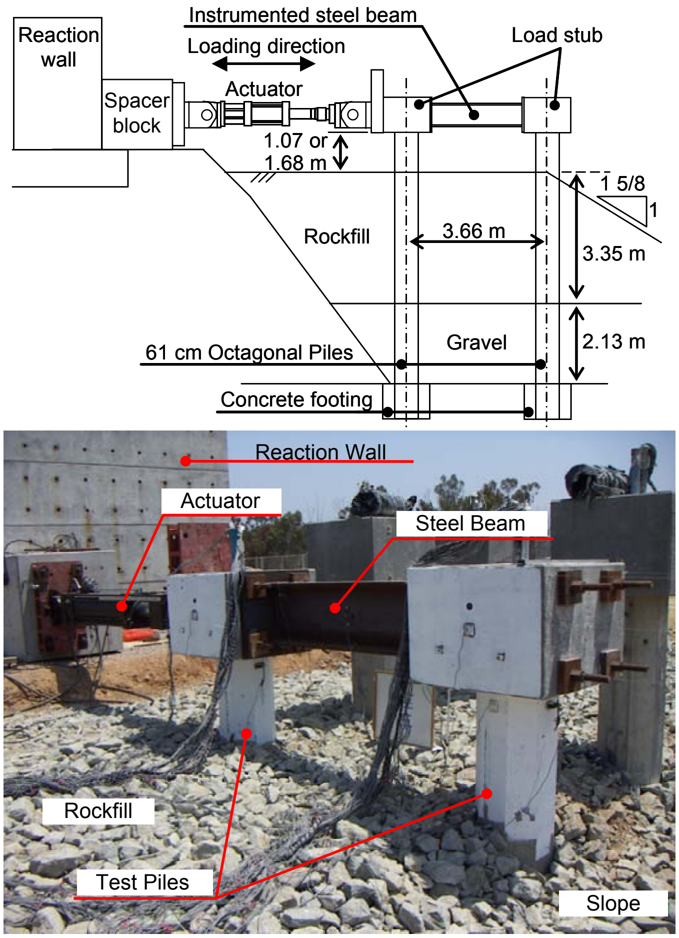


Fig. 3 Test setup – Coupled pile test

Obviously, it is best to use rockfills used for actual construction of wharves at POLA, however, such rockfill required large transportation costs. Therefore, it was decided to use similar material locally available in San Diego area. Smaller particle size gravel was used to fill the bottom 2.1 m of the pit as a working platform, and then the rockfill was placed on top of this gravel layer. The gradation of the rockfill used is shown in *Fig. 4* with typical POLA specifications. According to this figure, the rockfill material used in this series of experiments was more poorly graded than that typically used at the port. However, the gradation of this rockfill can be assumed reasonable because small particles may be washed out during placement of rockfill into the ocean, creating similar gradation conditions

To make a reasonable instrumentation plan, objectives and usage of sensor records have to be carefully considered. Because of the non-linearity of the prestressed concrete piles, it is difficult to define the bending stiffness of the test piles at any depth. Therefore, parametric analyses were performed to find *p-y* curves which provide reasonably fitted behaviors of the pile-soil system. Load-displacement curves at the pile top, profiles of rotation, deflection, and curvature and maximum moment location can be useful information for this back analysis.

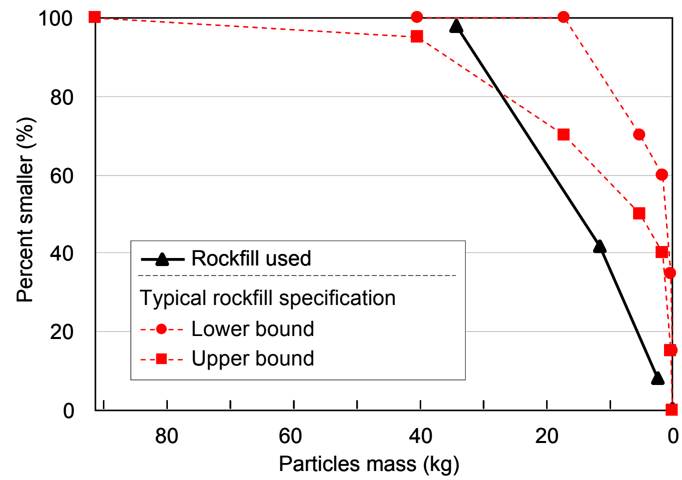


Fig. 4 Gradation of rockfill

Load and displacement at the pile top were recorded by sensors on the hydraulic actuator, and the load-displacement curves can be plotted from these records. Tiltmeters are one of the most powerful sensors to find profiles of pile behaviors. Data from a series of tiltmeters along the pile can be used to determine profiles of displacement and curvature as well as rotation. Tiltmeters, however, are not sensitive to local effects, such as cracking and spalling of surface concrete. To compensate for this disadvantage of tiltmeters, strain gages were also embedded in the test piles. Three independent gages were placed at various elevations so that curvature could be derived from these gages. Tiltmeters and strain gages were placed every one to two feet along the pile length, and more sensors were installed at the elevations where the critical section was expected.

More details on the test specimens, setup, and instrumentation plan can be found in Juirnarongrit *et al*. [2007].

OBSERVATIONS DURING AND AFTER TESTS

For the development of a reasonable hypothesis, it can be helpful to summarize significant observations and discuss them in detail. Notable observations during and after loading found in the entire series of tests are summarized in *Fig. 5*. *Fig .5*(a) through (d) were taken during in-ground inspection after loading, and *Fig. 5*(e) and (f) were taken after and during loading, respectively.

Fig. 5(a) shows fractured rock particles observed adjacent to the test pile in-ground. It implies that strong contact force working perpendicular to the contact particle surface broke the rock particles down. *Fig. 5*(b) shows “pock-marks” at the pile surface. These marks are an evidence of strong point load acting on the pile surface from the surrounding rock grains. Distribution of these pock-marks implies two important things; i) about 10 marks could be observed during inspection with 3 ft (0.9 m) excavation; i.e. the number of contacts was quite few, ii) some marks were found at some elevations, and

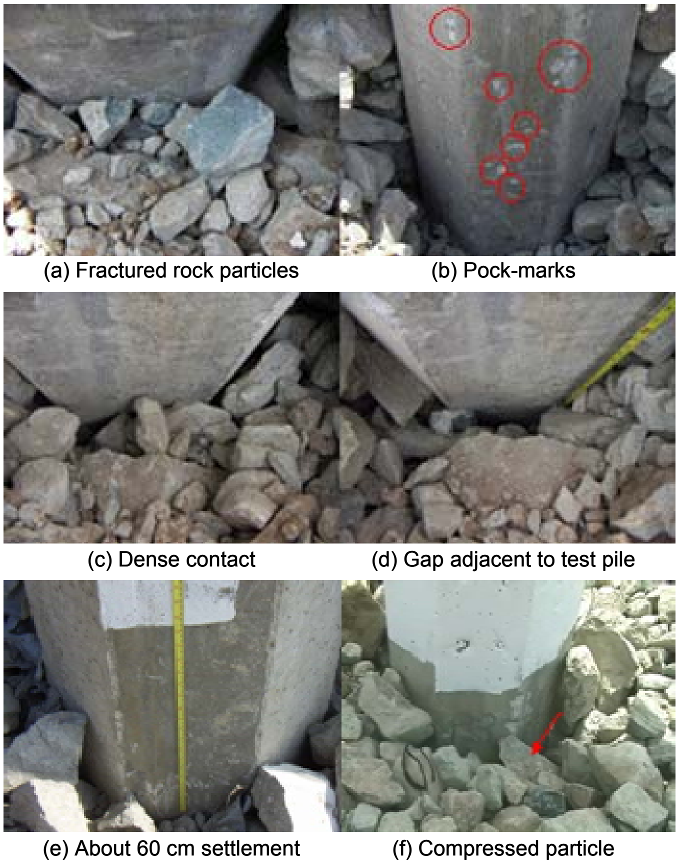


Fig. 5 Notable observations

no mark existed at the other elevations. It indicates distribution of contacts was random, and dependent on arrangement of the rockfill particles.

Fig. 5(c) and (d) show different density of contacts at different depth along the pile; contacts between the pile and the rock particles are relatively dense at some depths in *Fig. 5*(c), and quite loose at other depths in *Fig. 5*(d). Contact density and gap size may be significant factors to vary behavior of the pile systems.

Relatively large settlement of the rockfill immediately surrounding the piles was observed and is shown in *Fig. 5*(e). Although only portions above the original ground surface of the pile were painted before loading, the unpainted portions of the pile became exposed during loading. The maximum settlement was approximately 0.6 m. As an example, a sketch of settlement around P5, the pile at the dike crests in Coupled pile test 1, is shown in *Fig. 6*. It is notable that loading rate may be one of the factors because the settlement progressed quite slowly during loading and such a large settlement may not happen during higher rate loading, such as seismic motion.

Fig. 5(f) shows compressed rock particles between the test pile and adjacent rock particles observed during loading. Deflection of the pile was obviously larger than lateral movement of the adjacent rock particles. This particle compression may generate reactions even under significantly low vertical confinement. Also, it can be reasonably assumed

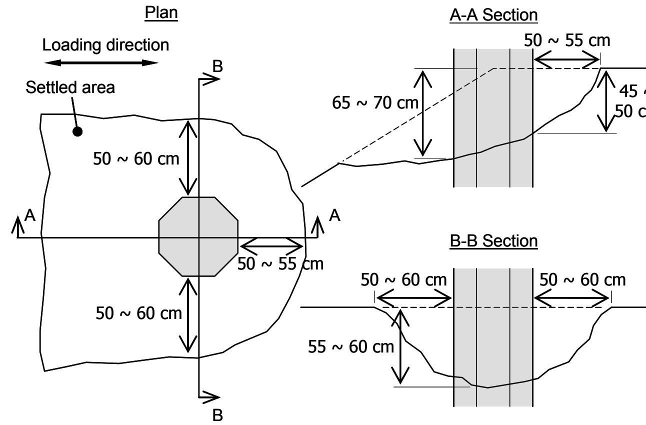


Fig. 6 Sketch of settlement around P5 (Coupled pile 1)

that the pock-marks at the surface of the test pile were developed at the contact points between the test pile and the compressed rock particles. In fact, Diaz *et al.* [1984] mentioned that the rock fill near the surface is stiffer than sand, and it may imply the rock fill generates stress independent reaction from structural interlocking. Also, McCullough *et al.* [2001] included “pseudo cohesion” in analyses to explain results recorded in the centrifuge tests.

TEST RESULTS AND ASSESMENTS OF P-Y CURVES CURRENTLY USED FOR DESIGN PRACTICE

In order to assess accuracy of current wharf structure design, numerical results obtained based on methodology and parameters used for current design practice are compared with test results recorded in this series of experiments. Because the objective of this comparison is assessment of the design *p-y* curves, pile and connection properties used herein were derived following the current design procedure. The *p-y* curves currently used for design are available in Martin [2005].

The property of the prestressed concrete pile used for analysis was simplified without degradation based on moment curvature relationship derived using commercial software, *XTRACT* [2007]. The property of the connection for Couple pile tests was defined based on the full-scale test conducted prior to this study (Krier 2006).

The settlement around the piles may play an important role on the behavior of the laterally loaded piles because the settlement around the pile makes pile length above ground surface longer. Therefore, the pile behaves more flexible, and the ultimate lateral resistance of the pile becomes less. The effect of the settlement around the pile on the load displacement relationship is shown in *Fig. 7*. In general, the pile behavior recorded during the test is between numerical results under initial condition without settlement and after loading condition with settlement. The effect of the settlement was included in the analyses only for the Coupled pile tests because the maximum input displacement at the pile top in the

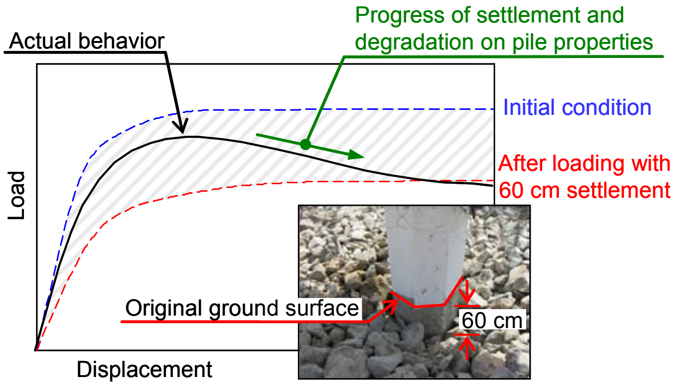


Fig.7 Conceptual comparison between numerical and experimental load displacement curves

Single pile test was not large, and significant settlement was not observed. Numerical analyses for cases with 0.6 m settlement were performed removing the *p-y* curves from the top 0.6 m.

Because of the lack of available information about laterally loaded pile systems in large particle rockfills, the current design code requires upper and lower bound analyses to cover all of possible uncertainties. In the analyses, 0.3 and 2.0 are recommended as for the *p*-multiplier; i.e. the *p-y* curves for the lower bound are 0.3 times of the curves for level ground while the *p-y* curves for the upper bound are 2 times greater than the curves for level ground. In this paper, only standard case and the upper bound analysis are shown for Single pile test, and the upper bound analyses with and without settlement are presented for Coupled pile test 1. Summary of the analysis cases is shown in *Table 2* with the *p*-multipliers (*mp*) used.

Based on readings of the sensors on the actuator, load displacement envelopes for loadings in both loading toward the downslope and the reaction wall are plotted in *Fig. 8* with the numerical results. The lateral loads in any direction are almost identical; i.e. the effect of the downslope was not obvious on the behavior of the pile because the pile may be located far (6 times of the pile diameter) from the slope crest. From this figure, it is obvious that the numerical results gave lower lateral resistance even in the upper bound analysis. Also, rotation profiles recorded on tiltmeter array along the pile and the numerical results are plotted together in Fig.9 at three different input pile top displacements. The numerical rotation profiles are reasonably fitted with the test results at any input displacement.

Load displacement curves recorded in Coupled pile test 1 are shown in *Fig. 10*. The system loaded toward the downslope showed less lateral load (1400 kN) than loaded toward the reaction wall (1600 kN). It indicates that the downslope decreased the lateral resistance of the system as well as lateral reaction against pile movement. As observed in the comparison of Single pile test results, the numerical results provided less lateral load than the test results, especially when the system was loaded toward the reaction wall. Again,

Table 2 Summary of analysis cases

Test Setup	*p*-multipliers for stress-dependent *p-y* curves		Settlement	Note
	Pile on level ground (m_{pl})	Pile on downslope (m_{ps})		
Single pile test	1.0	---	No	Standard
	2.0	---	No	Upper bound
Coupled pile tests	2.0	2.0	No	Upper bound w/o settlement
	2.0	2.0	Yes	Upper bound w/ settlement

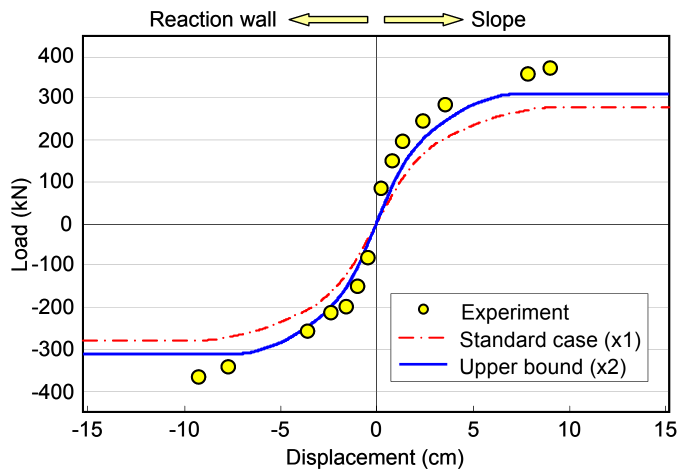


Fig.8 Load-displacement curves (Single pile test)

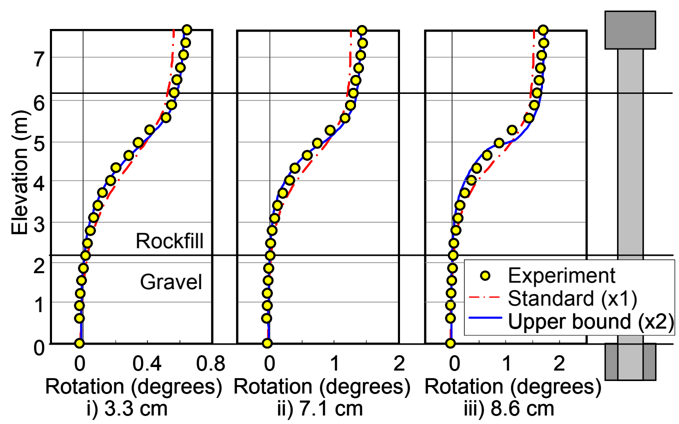


Fig.9 Rotation profiles (Single pile test)

recorded rotation profiles in Coupled pile test 1 are shown in *Fig. 11*. From this figure, the numerical results show reasonably similar rotation profiles as ones obtained in the test except for the pile at the crest when the system was loaded toward the downslope. It is because the upper bound analysis does not consider reduction of reaction due to downslope.

In addition, curvature calculated from strain gage readings along the test piles show another interesting phenomenon. The

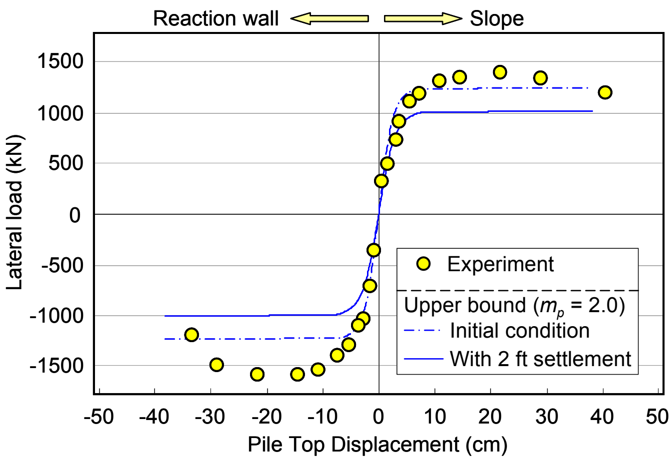
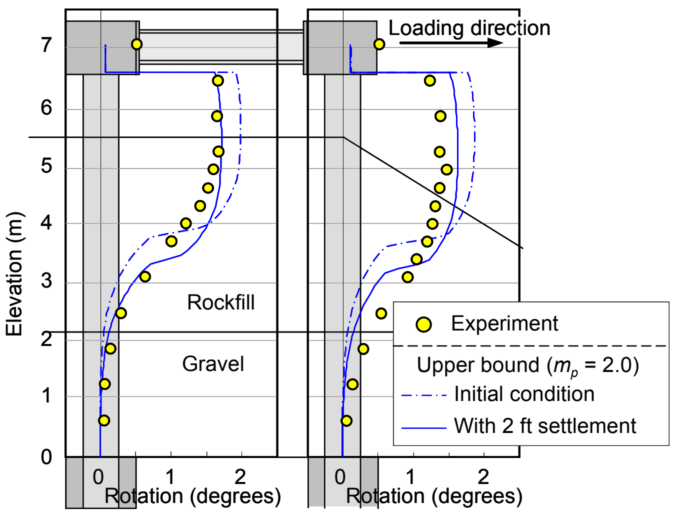
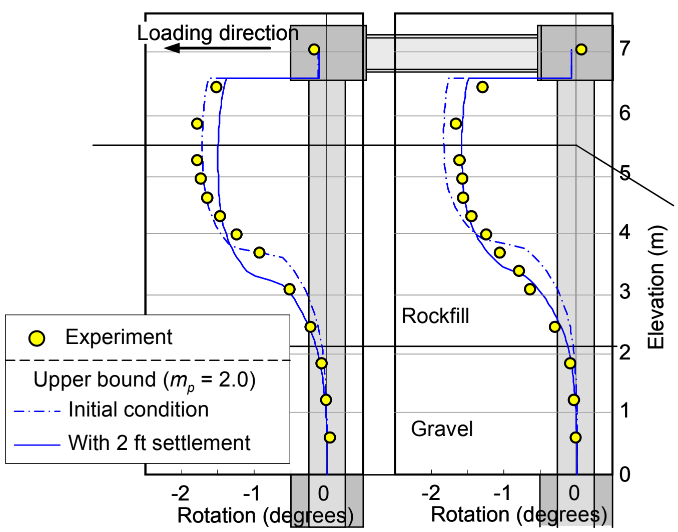


Fig. 10 Load-displacement curves (Coupled pile test 1)



(a) Loading toward downslope



(a) Loading toward reaction wall

Fig. 11 Rotation profiles (Coupled pile test 1)

curvature profiles of the pile at the crest in Coupled pile test 1 are shown in *Fig. 12* at three different input displacements of the pile top. The peaks of these curvatures indicate locations of local damage; i.e. cracking, spalling, and plastic hinge development. This figure shows that locations of the peak curvature shifted upward as input displacement became larger. This phenomenon is defined as migration of the critical section, and one of the possible mechanisms is shown in *Fig. 13*. As shown in Fig. 5(c) and (d), contacts between the pile and the large particle size rockfill were dense and strong at some elevations, and loose and weak at other elevations. Therefore, a zone with few contacts can exist along the pile (*Fig. 13*(a)). When the pile is loaded, the contact between the pile and the rockfill at deeper elevation starts generating strong reaction to the pile, and portions of the pile above that strong contact point deflect. Therefore, large curvature is developed and tensile cracks are generated at the elevation of the strong contact point. As the deflection of the pile becomes larger, the pile becomes in contact with another rockfill particle at shallower locations than the first strong contact point, and reaction to the pile at the new contact increases (*Fig. 13*(b)). When the reaction force at the newly generated contact point becomes large, deflection of the pile below the new contact point is restricted and the first crack ceases to expand. On the other hand, deflections begin to concentrate at the newly generated upper contact point and another cracking is developed there. This procedure of the migration can be repeated several times, making the critical section seem to shift upward as amplitude of the input motion becomes larger.

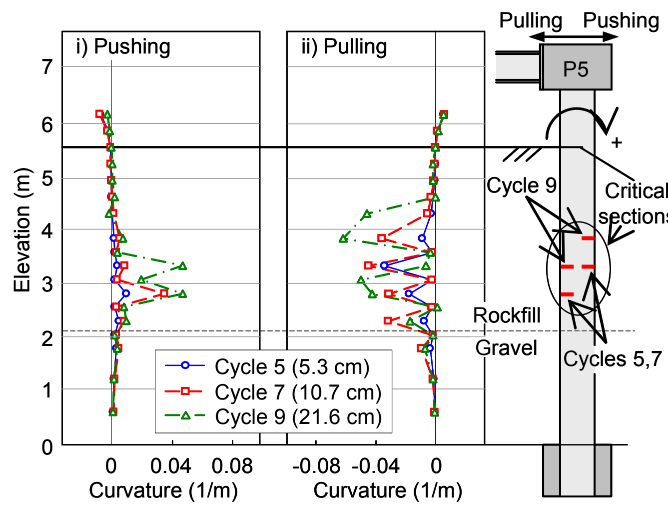


Fig. 12 Critical section migration in Coupled pile test 1

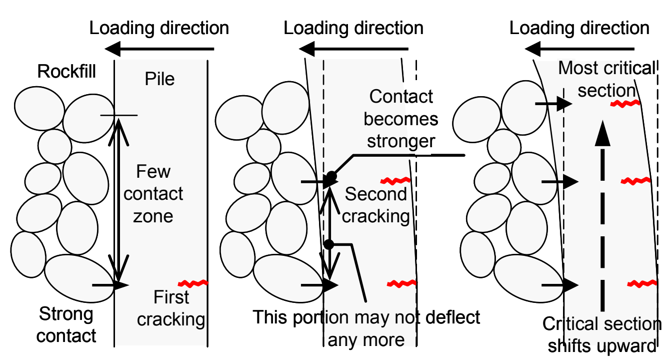


Fig. 13 Possible mechanism of critical section migration

POSSIBLE PARTICULATE MECHANISM AND MODIFICATION OF P-Y MODEL

Based on the aforementioned observations, possible patterns of particle movements are summarized in *Fig. 14*; i.e. a) translation or slippage, b) particle compression, c) rotation, and d) climbing over its neighbor particle related to dilation. In general, it is believed that the shear strength of cohesionless soil is composed of friction and dilation. A component of shear strength due to friction is generated during relative movement of particles, such as slippage and rotation. This component is dependent on effective vertical stress. Also, component from dilation is originated by the climbing over of soil particles, and its magnitude may be stress-dependent because vertical stress suppresses particle climb over. In addition, compression of particles shown in *Fig. 5*(f) is another origin of reaction. Reaction from the particle compression seems to be stress-independent and particulate structure-dependent and can be generated even under very low confining stress. Note that existence of particles staying in place is necessary for development of particle compression (*Fig. 14*(b)).

With the discussion above, a hypothesis can be developed; i.e. the lateral reaction from the rockfill to the piles is a combination of stress-dependent and stress-independent components. Because of the different stress dependency of those components, the *p-y* curves for those reactions need to be separately defined. For convenience, the stress-dependent reaction is defined as "friction" acting parallel to the contact surface between particles, and the other stress-independent reaction is as "particle compression" working perpendicular to the contact surface herein. Conceptual drawings of friction and interlocking are shown in *Fig. 15*.

Based on the above possible particulate mechanisms and past work (Diaz et al 1984, and McCullough 2001), it is concluded that stress-independent, "pseudo cohesion" representing particle compressions needs to be considered to improve accuracy of numerical analysis. In next chapter, impact of inclusion of particle compression concept on seismic design of wharf structures and dynamic behavior of large particle size rockfill are briefly discussed.

DISCUSSIONS ON DYNAMIC PROPERTIES OF LARGE PARTICLE SIZE ROCKFILL

Based on results from this series of cyclic static loading tests, a large amount of useful information is now available for displacement-based seismic design of wharf structures. To develop more sophisticated seismic design with dynamic motions, several factors need to be considered. In this paper, effects of loading type and rate are mainly discussed.

The seismic design code recently approved by POLA [2004] requires analyses for two different load conditions; i.e. inertial and kinematic load conditions. In an analysis for inertial load condition, lateral load due to inertial force of superstructures needs to be applied at pile tops, whereas lateral force along piles due to displacement of rockfill dike needs to be considered during analysis of kinematic load conditions. Based on the concept of the reaction due to particle compression, its intensity seems dependent on volume of compressed particles adjacent to piles. *Fig. 16* shows possible free body diagrams around pile for both the load conditions. It is notable that volume of particles in compression may be much smaller in force development for kinematic load than the other because rock particles around the pile can also move with free-field ground. Therefore, less pseudo cohesion may be expected in force development for kinematic load than the other. Because the pseudo cohesion obtained in this study is for inertial load conditions, effects of loading type must be carefully considered when pseudo cohesion needs to be applied in design for kinematic load condition.

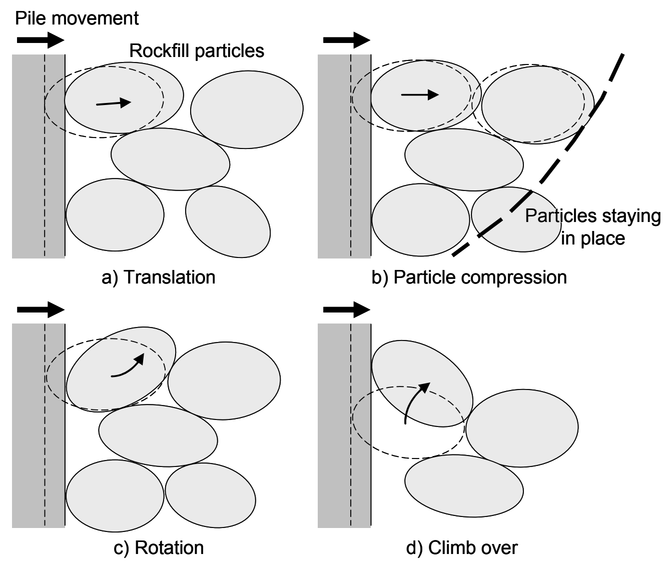


Fig. 14 Possible patterns of particle movements

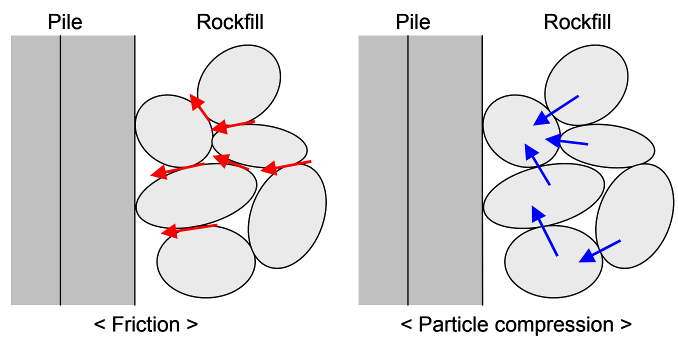


Fig. 15 Reaction due to friction and particle compression

Also, loading rate may affect pile-large particle rockfill interaction. Crushing strength and compressive stiffness of rock particles may be larger at higher late loading; i.e. rockfill has higher potential to generate larger reaction due to particle compression at higher rate dynamic loading. In addition, settlement around piles may be affected by rate of loading. As mentioned above, according to the observations of the settlement around the test piles in this series of experiments, the settlement of the rockfill progressed quite slowly. It implies that large settlement may not occur at higher rate loading. This could occur because rock particles have more opportunities to redistribute at lower rate of loading. Also, loading rate affects on dilation properties of the rockfills. Yamamuro and Lade [1993] stated "higher strain rates do not allow as much time for particle crushing and rearranging and this makes the soil appear less compressive or more dilatant, resulting higher strength" in sand. Because larger energy is required to produce dilation in larger particle size rockfill, it is likely that rock particles may be compressed more intensively at higher rate of loading, which results in a larger reaction due to particle compression. *Fig. 17* shows a possible relationship between loading rate, friction and particle compression with the definition that friction and particle compression are 1 at a very slow loading rate, assuming no crushing of the rock particles. Because smaller particles can redistribute or dilate at smaller displacement, they may also redistribute in shorter time. Therefore, the loading rate may be less significant on reaction due to friction and particle compression in smaller particle soil.

In addition, generally pore water pressure buildup during an earthquake in rockfill does not generate critical failure (i.e. liquefaction) because of its high permeability, but excess pore water pressures induced by earthquakes are not zero, especially when water is supplied from an inundated sea wave. It may reduce the ultimate reaction from the rockfill due to friction and particle compression, as well as effective normal stress.

SUMMARY

For the development of a more reliable design standard for pile-supported wharf structures in large rockfill, soil-pile-superstructure interaction is very important, but there is very limited amount of information available at present. In order to fill this knowledge gap, a series of full-scale tests and numerical analyses were performed. Based on observations during and after the tests, the *p-y* curves currently used for design were assessed. As a result, it was found that the *p-y* curves for current design practice gave much lower lateral resistance of the soil-pile system. It was also determined that the inclusion of the reaction due to the particle compression may improve the numerical results. Also, through discussions about effects of loading type and rate on pile-soil system behavior, the importance of additional research in future can be shown to quantify the hypothesis introduced in this paper and develop a more sophisticated seismic design.

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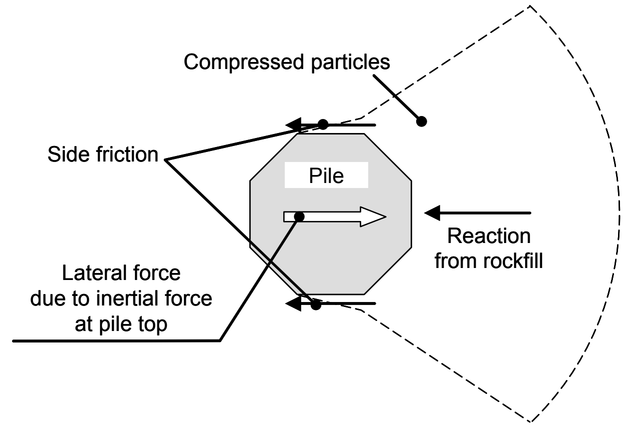
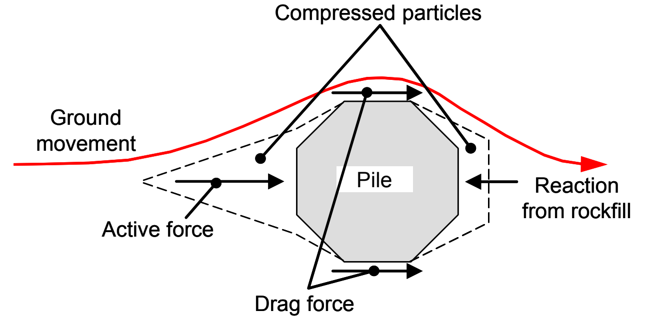


a) Inertial load condition



b) Kinematic load condition

Fig. 16 Free body diagrams around pile

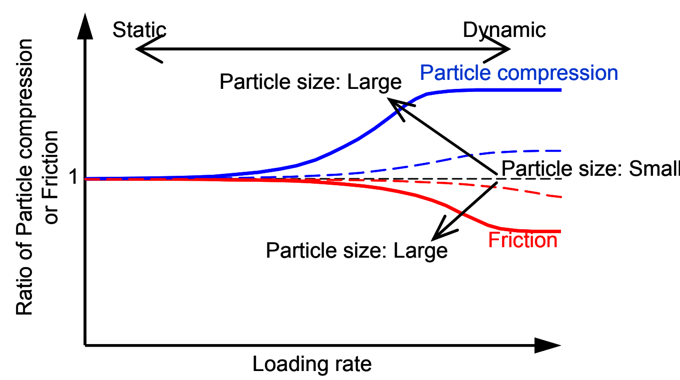


Fig. 17 Possible effects of loading rate

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